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Report on lateral load tests on mill cranes riveted crane no. 5886, 1940

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REPORT OF LATERAL LOAD TESTS ON MILL CRANES

RIVETED CRANE NO. 5886

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CRANE NO. 5886

I. SYNOPSIS

This report summarizes the results of lateral load tests on the bridge girders of a 60-ton riveted, fish-belly type steel mill crane of 120 ft. span. The girders were supported on four two-wheel equalizer trucks, and were tied together by a box end tie. Lateral load was applied to the top flange by taking up on a turnbuckle fastened to both cover plates. The stresses were measured on both end ties, on gage lines at each of seven sections, and on the walkway of one bridge girder. Both the stresses and sidewise deflection were measured when the load was applied. It was found that the whole cross section of the girder resisted the lateral load. However, at the center section, secondary torsion stresses increased the top flange stresses and decreased the bottom flange stresses an average of about 25 per cent. End fixity was also present, and the fixed end moment was 84 per cent of that for a fully fixed beam. This end fixity reduced the lateral load stresses 37 per cent, and the lateral deflection 58 per cent. It is thus very important in maintaining the stiffness of the crane. The load applied was approximately equal to the design lateral load due to all causes.

II. INTRODUCTION

This test is one of a series being made on various types and proportions of bridge girders to determine how much of the section of the girder is effective in resisting the lateral loads. The large resistance of box girders to twisting would indicate that the whole section of the girder resists the lateral bending and not only the top flange as is usually assumed. This is also the conclusion from the theory. To determine this, tests were made on Crane No.5886 in the fabricating shop. The end tie was partially bolted to the girders and most of the remaining holes were filled with drift pins. The whole assembly was supported on the end trucks. The stresses were measured along the bridge girder, walkway and end tie; and knowing the stresses, the section which resists the lateral moment is determined. A sketch showing some of the details of the girder is shown in Fig. 1.

The stresses were measured by means of a 10-in. Whittemore Strain Gage. Gage lines were established at the various sections of the girder where the stresses were desired, by drilling two very small holes in the girder ten inches apart. The distance between the holes was measured with the gage before and after the load was applied. The difference in length between these two readings gave the strain in the member due to the applied load. The results are generally accurate to ±300 lb per sq in.

The lateral deflection of the girders was also found by measuring the change in distance between the adjacent flanges of the two girders. This was done on the top and bottom flanges at seven sections along the girder. A Federal Dial measuring the distance between the girders to 0.001-in. was used to determine this deflection and the difference in readings between the initial and final loads gave the actual movement of the flanges. The difference between the lateral deflection of the top flange and bottom flange at the same section gave the twist of the girder at that section. The twist in inches divided by the depth of the girder at that point gives the twist in radians. Knowing the twist, one can determine the lateral deflection of the girder as a whole.

The load was applied by tightening a turnbuckle fastened to the two girders. A calibrated spring was interposed in the system and its closure was measured by a 0.001-in. Ames dial. The closure was a measure of the actual load. A lateral load of 9300 lb. and 20,000 lb. was applied to the girder. The 20,000-lb. load is approximately the design lateral load.

III. TEST RESULTS

Fig. 2 shows the lateral deflection curves of the top and bottom flanges of the bridge girders. The measured values were equal to the deflection of both girders and the plotted values are the measured values divided by two since the deflection of both girders should be equal. The deflections are plotted along the length of the girder and the center line mark denotes the center line of the bridge girder.

It should be noted that the end deflection is not zero for the bottom. This is due to initial slip in the joint. The slip is larger at the bottom than the top because the girder hangs below the end tie for some distance. The girder thus swings about a pivot point in the end tie and consequently the movement on the bottom is quite large.

The difference between the deflection readings of the top and bottom flanges at the same section gave the twist of the girder at that section. This is plotted in angular measure in Fig. 3. It was rather surprising to note that the twist was practically constant along the length of the girder. The measured twist was about 0.00160 radians or 0.09-degree. This includes the end slip. If the end connection had been fully bolted, this value would undoubtedly have been smaller. However, in service, the end connection could quite probably loosen enough to allow this end play. The theoretical twist for this girder is 0.00119 radians and this assumes no slippage. However, the theoretical value should be reduced to take into account the fact that torsional warping is prevented at the middle and ends of the crane, and this reduces the actual elastic twist over the computed theoretical.

The lateral deflection of Fig. 2, corrected for initial slip and for the twist shown in Fig. 3, is given by Fig. 4.

The stresses were measured at seven sections along the girder and also on the center section of the end truck. The coverplate readings were taken on the edges. Readings were also taken along the center line of the web. Fig. 5 shows the stress distribution at the center section of the girder, and the plotted points show where the gage lines were located. The stress is plotted from the adjacent side of the box section as an origin. The center section was very close to a diaphragm.

In Fig. 6 is shown the variation of stress along the girder on the inside and outside edges of the top coverplate. Fig. 7 shows the stress distribution along the web, and Fig. 8 shows the stress distribution along the bottom coverplate. The theoretical stress distribution is not a straight line because the section of the girder varies. In Fig. 6, the stress at the center was greater than the theoretical, and in Fig. 8, the measured center stress was less than the theoretical. The rest of the girder checked fairly well with the theoretical. The average center variation was about 25 per cent. This variation is probably due to secondary torsional stresses at the center of the crane. Since the direction of twist reverses at the bridge center there can be no warping due to twist at this section. It is not likely that the stress variation is due to the top flange taking an appreciably greater proportion of the lateral load than the bottom flange, because the difference between the deflections of the top and bottom flanges is a much smaller percentage than the measured variation in stresses.

The point of zero stress is not at the end of the girder but about 22 ft. in from the end of the girder. This corresponds to a fixity factor of 84 per cent. The statement 84 per cent partial fixity means that the moment at the end of the bridge girder is 84 per cent of that which would be present if the girder acted as a fully-fixed beam. It should be remembered, however, that no attempt is usually made to take advantage of the end fixity in design, and the girders are designed as simple beams.

The theoretical stresses shown in the diagrams were computed using the end fixities found from the points of contraflexure, and using the gross moment of inertia at the section considered. The gross moment of inertia is used since stress measured by a Whittemore strain gage usually checks the theoretical computed on the basis of the gross moment of inertia.

As an additional check, the lateral deflection was computed using the equations in the appendix. From Fig. 4, the measured deflection under the 20,000-lb. load is seen to be 0.90 in. The computed deflection was 2.10 - 1.21 = 0.89 in. which is a good check. The value of 2.10 in. is the deflection which would take place if there were no end fixity. The 1.21 in. is the deflection due to the end moment and acts in the minus direction. The difference is the actual deflection. The importance of end fixity and of maintaining the end tie tight is apparent if a stiff crane is desired.

The walkways on this crane seemed to have very little effect on the deflection; as the computations above were made disregarding the walkway. Strain measurements on the walkway also showed no stress.

The formulae used in the computations are given in the appendix.

IV. CONCLUSIONS

1. The whole section of the box girder resisted the lateral moment.

2. Partial end fixity was present which reduced the lateral deflection over fifty per cent. The fixity factor was 84 per cent.

3. Secondary torsion stresses were present which increased the top flange stresses and reduced the bottom flange stresses.

4. The walkway carried no stress and had little effect on the deflection of the girder.

V. APPENDIX

The following formulae were developed for fish-belly girders from the fundamental differential equation

$$E\,\frac{d^2y}{dx^2} = -\frac{M}{I} \qquad (1)$$

Since I is a variable in this equation as well as M, some assumption must be made as to the variation of I. Two sets of formulae were derived, one in which it was assumed that I varied linearly along the girder, and second, it was assumed that I varied parabolically along the length of the girder. The first approximation is most applicable to computing the lateral moment of inertia, and the second in computing the vertical moment of inertia. For average girders they should give results which are fairly close.

The computations for deriving these formulae are quite laborious and only the results will be given.

For a girder in which the I varies parabolically, the deflection at any point x due to a load at the center of the girder is

$$d = \frac{-Pl^2}{16EI_3}\left[x \log\left(\frac{I_1 l^2}{I_3 4} + x^2\right) - 2x + 2\sqrt{\frac{I_1 l^2}{4I_3}}\left(\tan^{-1}\frac{x}{\sqrt{\frac{I_1 l^2}{4I_3}}}\right) - x \log\left(\frac{I_1 l^2}{I_3 4} + \frac{l^2}{4}\right)\right] \qquad (2)$$

and for the center deflection, this reduces to

$$d_c = \frac{-Pl^3}{16EI_3}\left(-1 + \sqrt{\frac{1}{\frac{I_2}{I_1} - 1}}\,\tan^{-1}\sqrt{\frac{I_2}{I_1} - 1}\right) \qquad (3)$$

P stands for the center load in pounds, l is the span in inches, I_1 is the gross moment of inertia at the end, I_2 is the gross moment of inertia at the center, I_3 equals $I_2 - I_1$, and E is the modulus of elasticity. The above formulae give the deflection for a simple beam.

The corresponding formulae for linear variation of I are

$$d = \frac{-Pl}{8EI_3^2}\left[I_3\,x(x-l) + I_1\,xl\,\log\left(\frac{I_1 l + I_3 l}{I_1 l + 2I_3 x}\right) - \frac{I_1^2 l^2}{2I_3}\log\left(1 + \frac{2I_3 x}{I_1 l}\right) + I_1 x l\right] \qquad (4)$$

and for the center deflection

$$d_c = \frac{-Pl^3}{16EI_3}\left[-\frac{1}{2} + \frac{I_1}{I_3} - \left(\frac{I_1}{I_3}\right)^2 \log\left(1 + \frac{I_3}{I_1}\right)\right] \qquad (5)$$

The deflections due to the end moment are given by the following formulae where M is the end moment.

For parabolic variation of I

$$d = \frac{-Ml^2}{4EI_3}\left[\frac{2x}{l}\sqrt{\frac{I_3}{I_1}}\,\tan^{-1}\frac{2x}{l}\sqrt{\frac{I_3}{I_1}} - \frac{2x}{l}\sqrt{\frac{I_3}{I_1}}\,\tan^{-1}\sqrt{\frac{I_3}{I_1}} - \frac{1}{2}\log\left(1 + \frac{4I_3x^2}{I_1 l^2}\right)\right] \qquad (6)$$

and for the center deflection, this reduces to

$$d_c = \left(\frac{Ml^2}{8EI_3}\right) \log \left(1 + \frac{I_3}{I_1}\right) \tag{7}$$

For linear variation of I

$$d = -\frac{Ml}{2EI_3}\left[x \log\left(\frac{I_1 l + 2I_3 x}{I_1 l + I_3 l}\right) - x + \frac{I_1 l}{2I_3} \log\left(\frac{I_1 l + 2I_3 x}{I_1 l}\right)\right] \tag{8}$$

and for the center deflection, this reduces to

$$d_c = \frac{-Ml^2}{4EI_3}\left[-1 + \frac{I_1}{I_3} \log\left(1 + \frac{I_3}{I_1}\right)\right] \tag{9}$$

The fixed end moment is given by the following formula for linear variation of I. There would be no occasion to compute it in the vertical direction.

$$M = \frac{Pl}{4}\sqrt{\frac{I_1}{I_3}}\;\frac{\left[-1 + \frac{I_1}{I_3} \log\left(1 + \frac{I_3}{I_1}\right)\right]}{\tan^{-1}\sqrt{\frac{I_3}{I_1}}} \tag{10}$$

The actual fixed end moment would vary from sixty to ninety per cent of the above value depending on the end connection.

In the above equations; the units must be consistent, and it is advisable to use pounds and inches. The angles are expressed in radians and the logarithms are natural logarithms to the base e and not to the base 10.

These equations require a good deal of computation for their solution, except for the deflections at the center where they are considerably simplified. The effect of varying the moment of inertia is to increase the deflection over what would take place if the whole girder section were equal to that at the center. For a number of girders which have been checked by the writer, the increase in deflection has varied from ten to thirty per cent. Therefore for approximate computations, the following formula is suggested for the center deflection.

$$d_c = K \left[\frac{Pl^3}{48EI_c} - \frac{Ml^2}{8EI_c} \right] \qquad (11)$$

I_c is the moment of inertia at the center, M is the fixed end moment which will vary from 60 to 85 per cent of the fully fixed beam depending on the connection, and K is a constant to take into account the fishbelly effect. K varies from 1.10 to 1.30, the smaller value applying to relatively flat fishbelly girders and the higher value to deep fishbelly girders. For a girder of constant section K equals 1.00.

In a fishbelly girder, the twisting moment due to a lateral load applied at the top flange is not constant but varies along the girder since the center of gravity of the girder approaches the top flange as one approaches the end.

The twist for this condition is given by the following formula, when the depth of the girder varies parabolically.

$$\gamma = \frac{Pl}{16b^2G}\left[\frac{b\,(t_2 + t_1)}{2t_1t_2\sqrt{h_1h_3}}\tan^{-1} x\left(\frac{2}{l}\right)\sqrt{\frac{h_3}{h_1}} + \frac{2x}{lt_3}\right] \qquad (12)$$

and the twist at the center

$$\gamma_c = \frac{Pl}{16b^2G}\left[\frac{b\,(t_2 + t_1)}{2t_1t_2\sqrt{h_1h_3}}\tan^{-1}\sqrt{\frac{h_3}{h_1}} + \frac{1}{t_3}\right] \qquad (13)$$

γ is the angular twist in radians, b is the distance between the web centers, G is the shear modulus, t_1 and t_2 are the cover plate thicknesses, t_3 is the web thickness, h_1 is the depth of web at the end, h_2 is the center web depth, and h_3 is the difference between the center web depth and end depth.

However, for approximate computations, we may use for the center twist

$$\gamma_c = \frac{Pl}{32b^2h_2G}\left[\frac{b}{t_1} + \frac{b}{t_2} + \frac{2h_2}{t_3}\right] \qquad (14)$$

This formula is nothing but the formula for the uniform section, and its use is based upon the assumption that the resistance to twist and the applied moment at any section vary proportionately. For the crane tested, the computed value given by equation (14) is practically the same as that given by the more exact equation (13).

The twist caused by a uniform torque along this girder, such as the motor torque may be computed by the following formula.

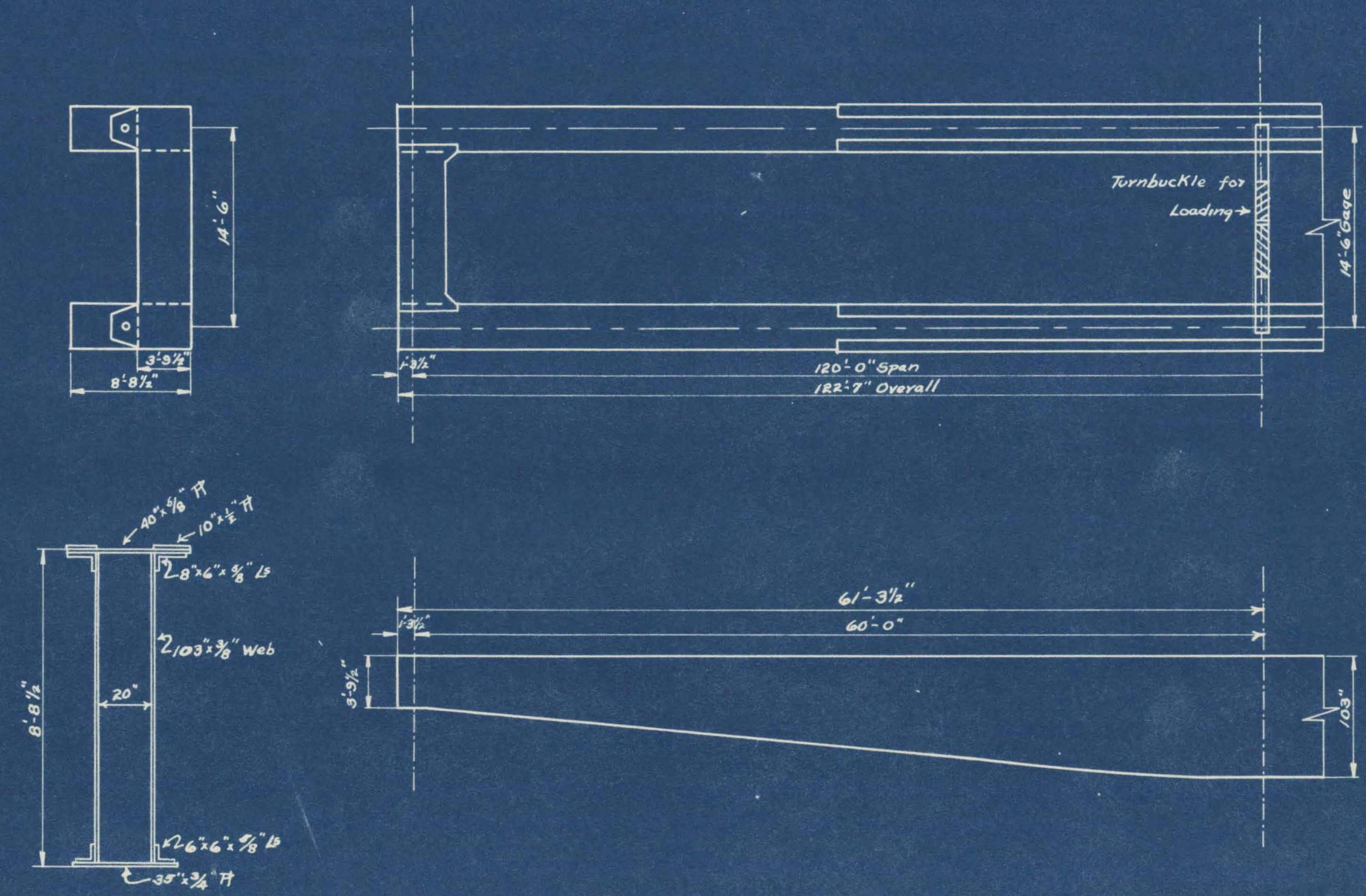
$$Y = \frac{Ml}{4Gb^2}\left[\left(\frac{b}{t_1}+\frac{b}{t_2}\right)\frac{lx}{2h_1(h_1l^2+4h_3x^2)}+\left\{\left(\frac{b}{t_1}+\frac{b}{t_2}\right)\frac{1}{4h_1}+\frac{1}{t_3}\right\}\frac{1}{\sqrt{h_1h_3}}\tan^{-1}\frac{2x}{l}\sqrt{\frac{h_3}{h_1}}\right] \quad (15)$$

and the twist at the center is simply obtained by substituting $x = \frac{l}{2}$ in the above formula.

The above formulae are given here as a matter of record since it was necessary to derive them in order to see how the test results checked. Their application in practice would probably be laborious. As a matter of fact, a properly designed box girder is so stiff that there is little necessity for computing twist.

The formulae which might have a practical application are those for lateral deflection since they are a measure of the stiffness of the crane. Also, an average crane often has a lateral deflection under the design load which is greater than the vertical deflection. Since this lateral deflection occurs in both directions, there can be no camber; and the resultant wear on shafting, bearings, etc. is more serious than that due to the vertical bending.

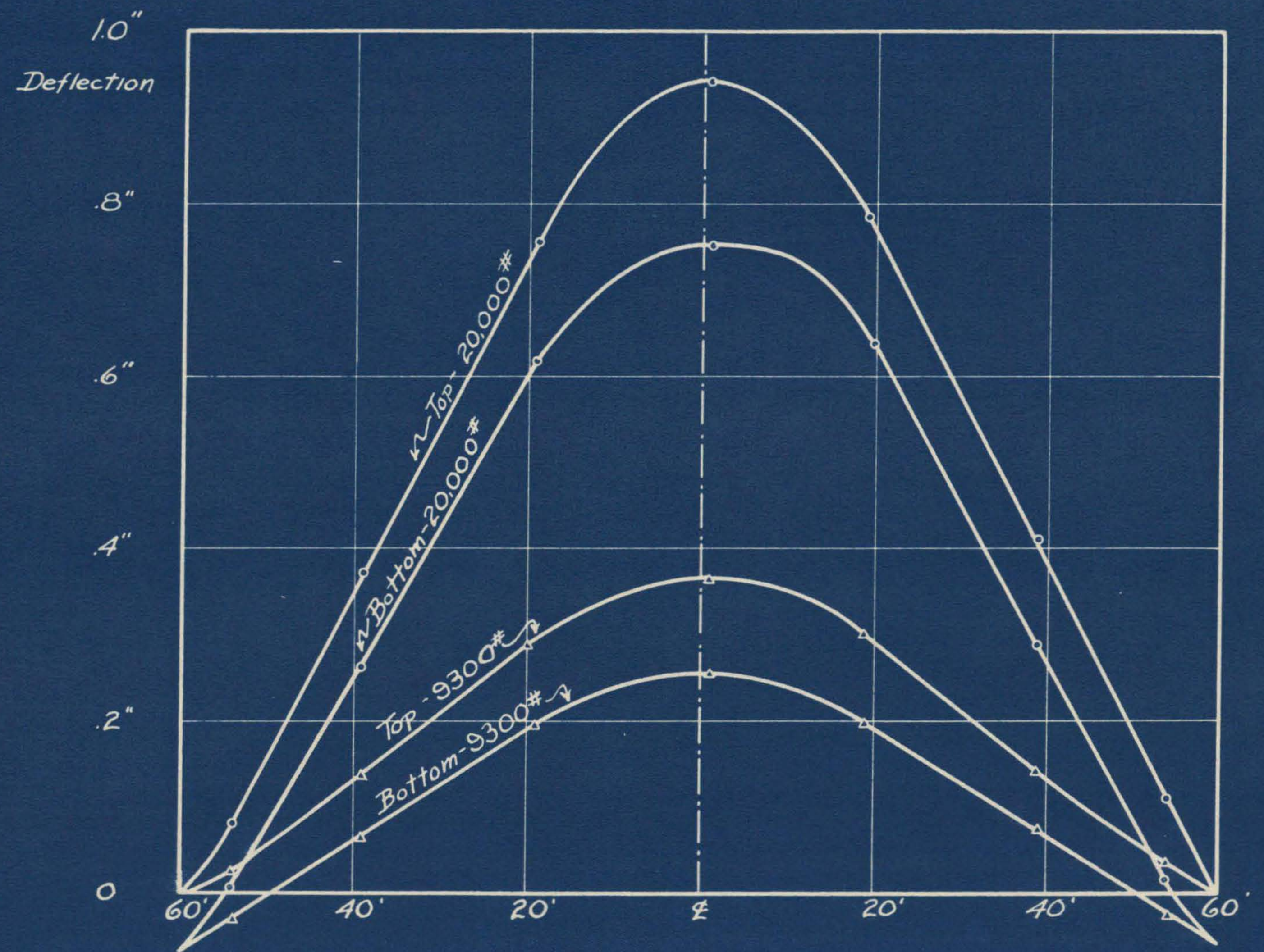
The above equations apply when the girders are loaded at the center, and this condition gives the maximum possible deflection very closely.



Section at ℄ Girder

Sketch of Crane #5886

FIG. 1



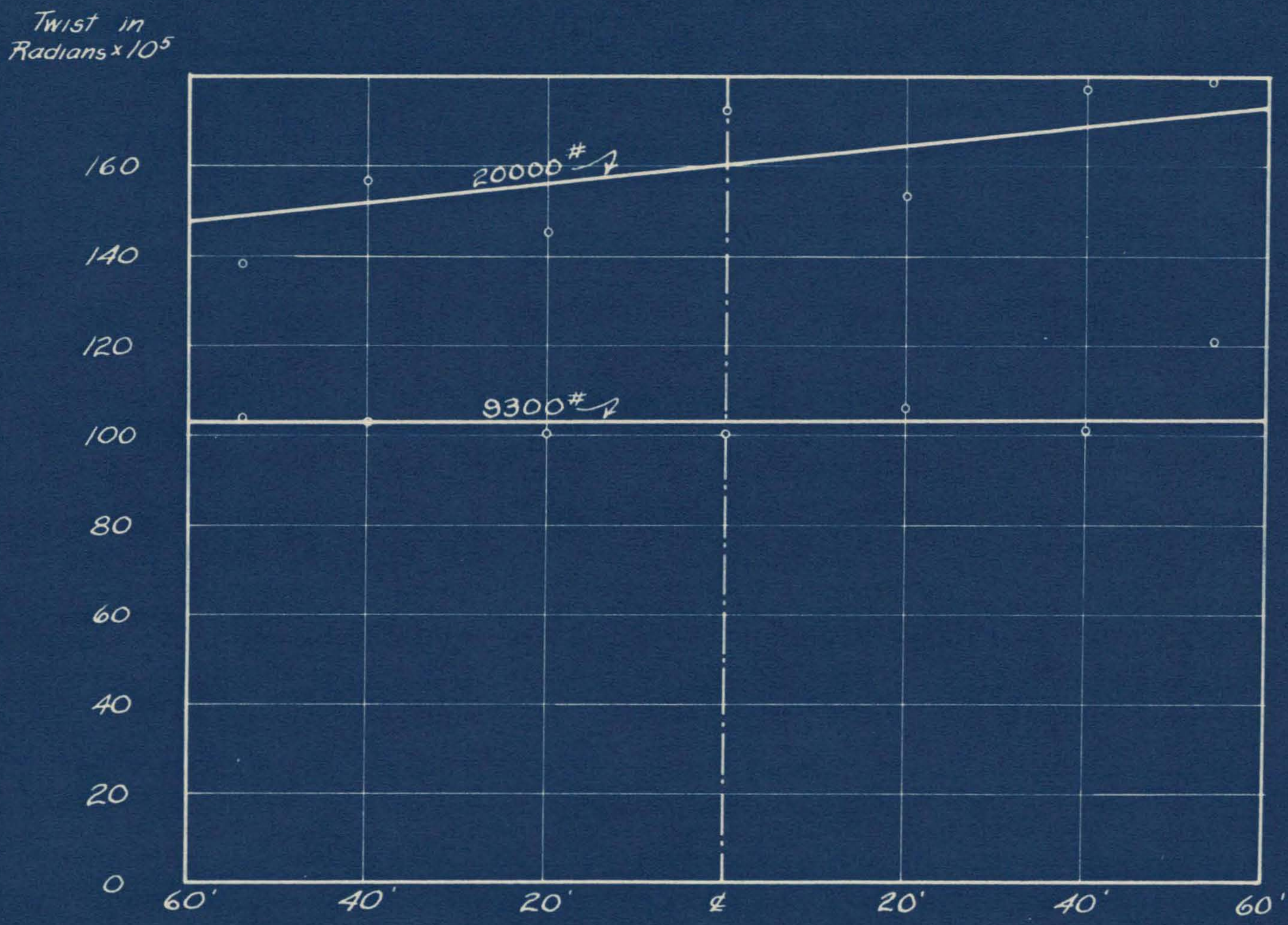
LATERAL LOAD TEST

CRANE #5886

DEFLECTION OF GIRDER

Nov. 5, 1940

FIG. 2



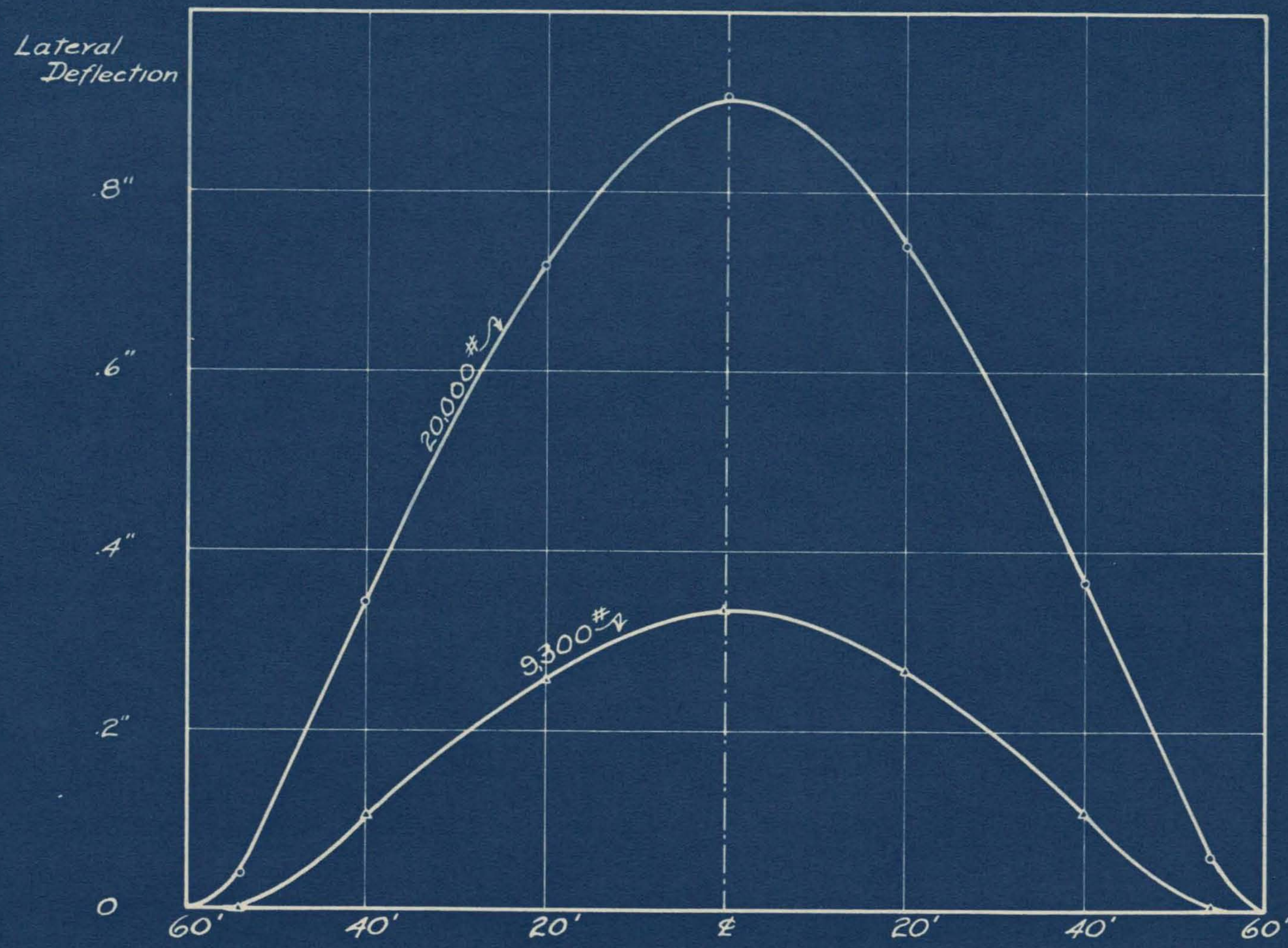
LATERAL LOAD TEST

CRANE #5886

TWIST OF GIRDER

Nov. 5, 1940

FIG. 3



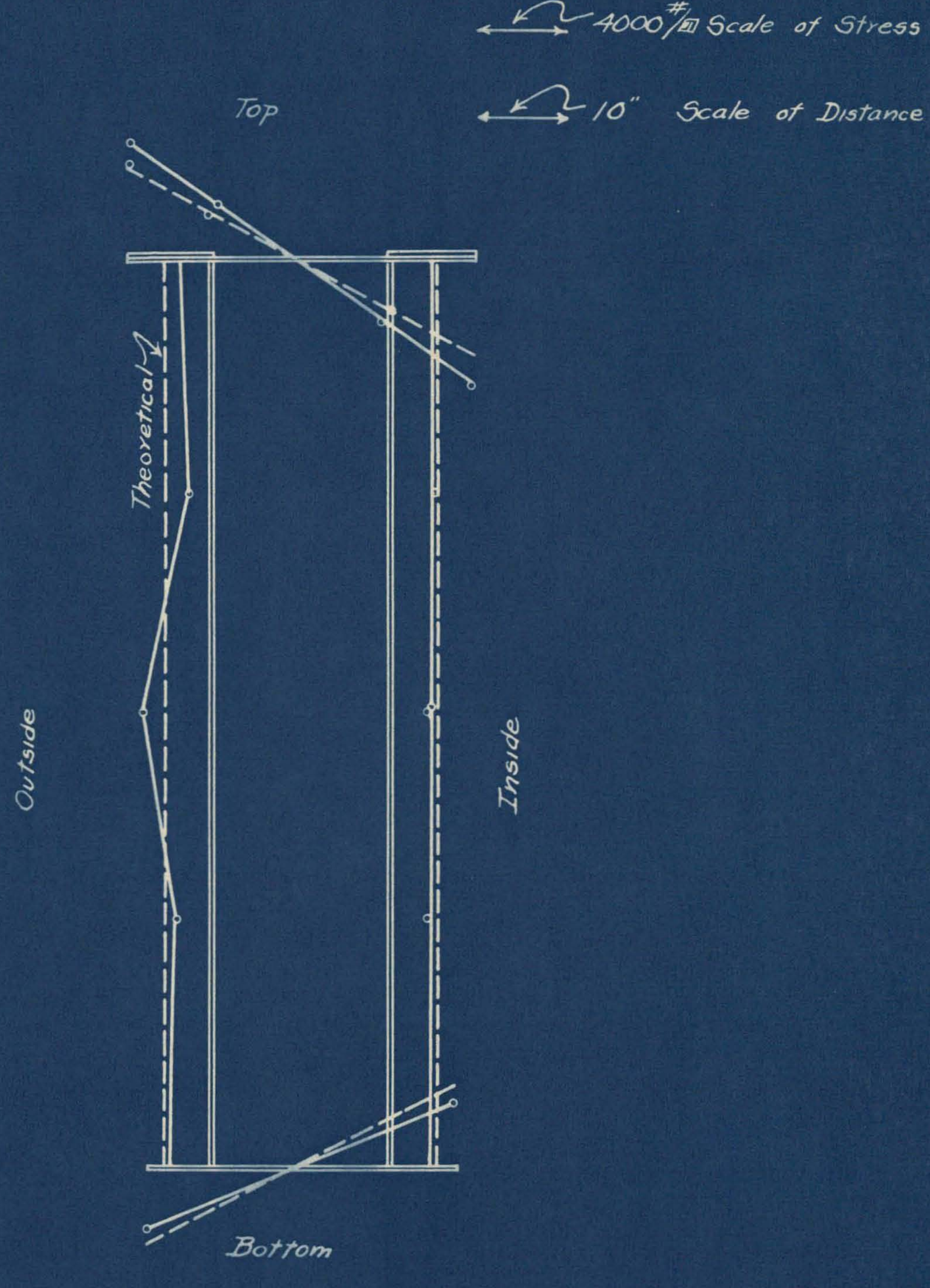
LATERAL LOAD TEST

CRANE #5886

DEFLECTION OF GIRDER

Nov. 5, 1940

FIG. 4



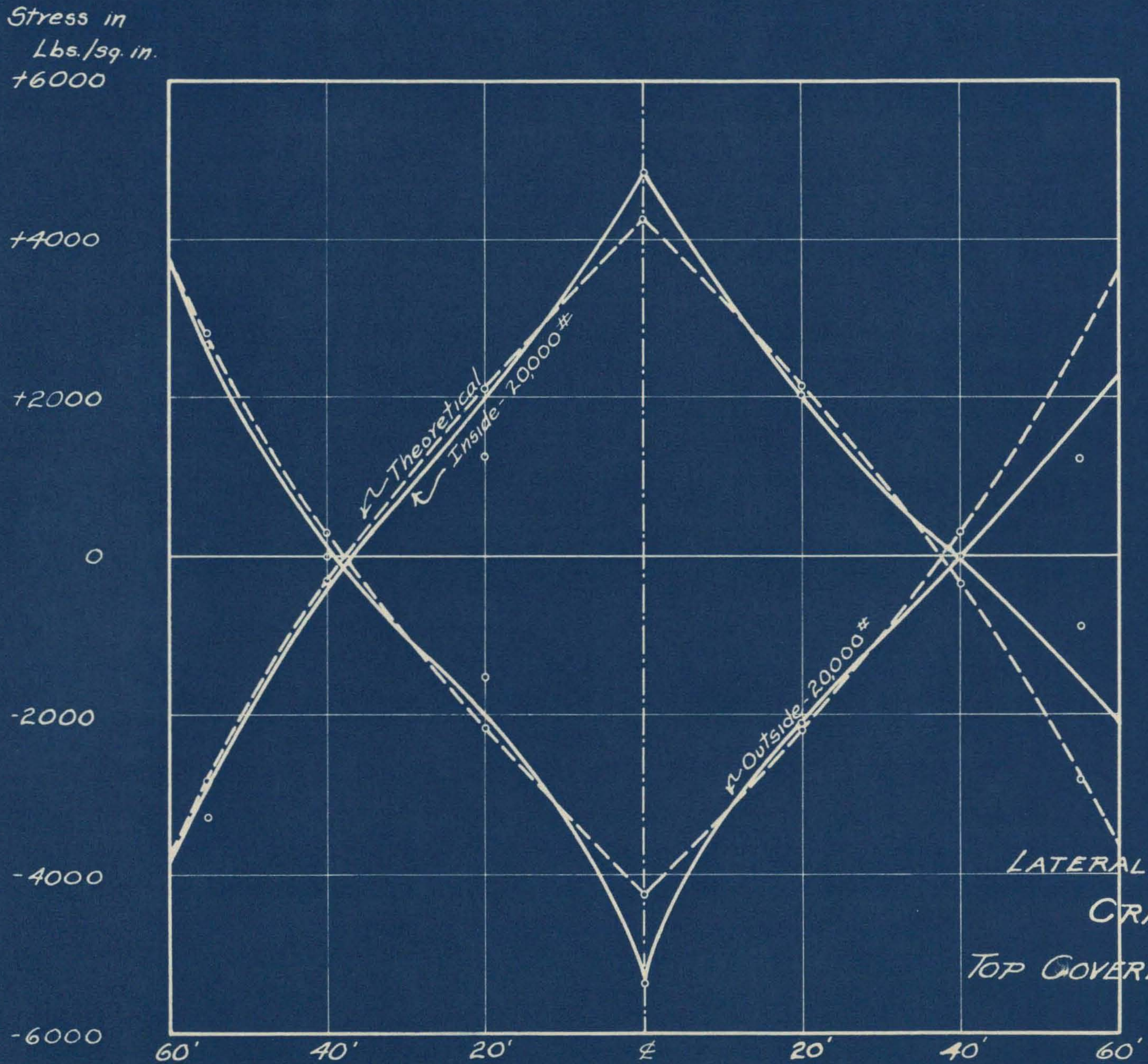
LATERAL LOAD TEST

CRANE #5886

STRESS AT ℄ OF GIRDER

Nov. 5, 1940

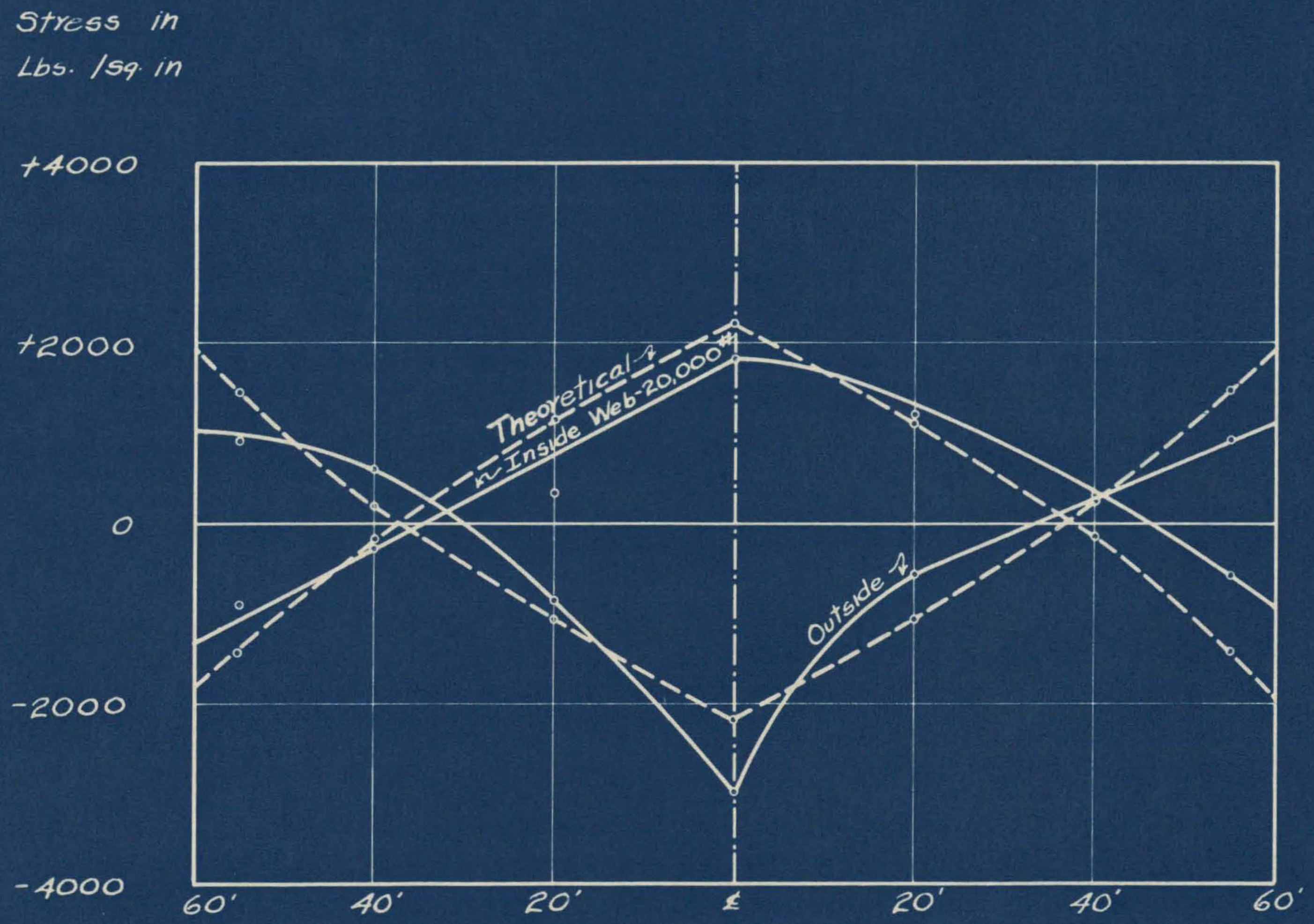
FIG. 5



LATERAL LOAD TEST
CRANE #5886
TOP COVERPLATE STRESSES

Nov. 5, 1940

FIG. 6



LATERAL LOAD TEST

CRANE #5886

WEB STRESSES

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FIG. 7

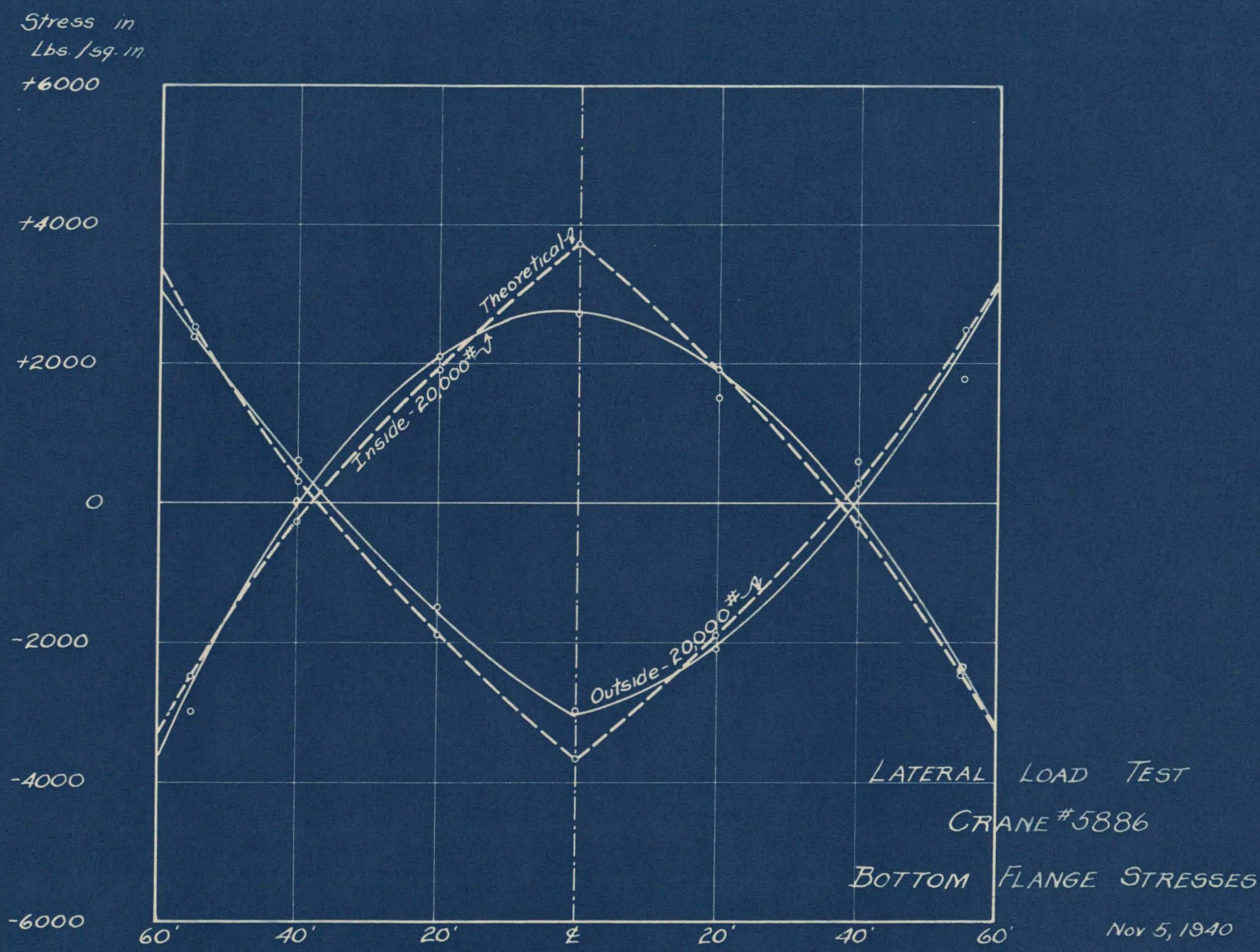
Stress in
Lbs. /sq. in.
+6000
+4000
+2000
0
-2000
-4000
-6000
60'
40'
20'
℄
20'
40'
60'
Theoretical ℄
Inside - 20,000# ℄
Outside - 20,000# ℄
LATERAL LOAD TEST
CRANE #5886
BOTTOM FLANGE STRESSES
Nov 5, 1940

FIG. 8